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Modelling of connections and lateral behavior of high-rise modular steel buildings

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ABSTRACT

Prefabricated Prefinished Volumetric Construction (PPVC), which is a form of modular construction, has been promoted recently for high-rise buildings to raise construction productivity. The flexibility of the inter-module connections and discontinuity of floor slabs of individual modules need to be modelled correctly in the structural analysis since they have direct effect on the building stiffness and its corresponding responses under lateral loads. In this paper, translational spring models are proposed to model the load transfer behavior of the vertical modules connections which are crucial for the structural behavior of high-rise modular buildings. The accuracy of the proposed spring models is investigated by comparing the force distribution and load displacement behavior of modular braced frames with conventional frame model established based on assumptions that the beams are either pin or rigidity connected to the columns. To enhance the productivity and work efficiency of high-rise modular construction, the feasibility of connecting the modules at the corners rather than tying the abutting beams or slabs is proposed. A more realistic approach of modelling the floor slab consisting of multiple modules inter-connected at the corners is recommended. The effectiveness of the corner connected modules in transferring the horizontal forces to the building's lateral load resisting systems is evaluated.

1. Introduction

The adoption of Prefabricated Prefinished Volumetric Construction (PPVC) is becoming more popular in highly populated cities like Singapore, where shortage of workforce and urban land supply are critical [1]. It is a form of modular construction where the internal elements of volumetric modules such as walls, floors, ceilings, furnishing etc., are prefinished in factories prior to assembly at site, as shown in Fig. 1 [2]. The first completed PPVC project, the Crowne Plaza Changi Airport Hotel Extension, utilized 252 steel PPVC modules to construct a 10-storey building. This project demonstrated a gain of construction productivity with labour and time savings of up to 50%. Despite various advantages in the application of modular construction, many industry players are still reluctant to be the "first movers" to fully utilize such technology [3]. Keys challenges in the adoption of modular construction involves the design, construction and project management aspects which are significantly different from that of a conventional construction method for a building [4–7].

Generally, the modular units can be fabricated using two different systems; load bearing module and corner supported module [2,8–11]. For load bearing module, load bearing walls are used to resist the gravity and lateral forces whereas for the corner supported modular system, it consists of a beam-column framing system. The present study focused on the corner supported modular structural framing system, as shown in Fig. 2. It is faster to construct a multi-storey building by connecting the modules at the corners. Such module has better structural performance-to-weight ratio and higher flexibility in architectural layout plan for modular units [9]. The corner supported modular units are designed to resist the gravity loads whereas an independent reinforced concrete core wall system is needed to resist the lateral forces. For tall buildings, the ease of connecting the modules affects the speed of construction and the rigidity of the inter-module connections affects the overall stability of the building. The modular building consists of separate modular units stacked up vertically and horizontally as illustrated in Fig. 2. For corner supported modular system, the modules are connected to one another at the corners, although they can also be connected through floors or along the edge beams. Due to the geometric constraints where multiple beams and columns meet at the connecting regions as well as the obstruction of walls and internal finishes (which are completed in factory), novel connections have to be developed to connect these modules [9,12,13]. These connections have direct effect on the building stiffness and its corresponding responses under lateral loads [14–17].

Research has been conducted to study the performance of inter-module connections. Novel inter-module joints utilizing mechanical fasteners such as bolts, shear keys, threaded rod, tie plates, etc., were proposed. The inter-module connection can be categorized into two

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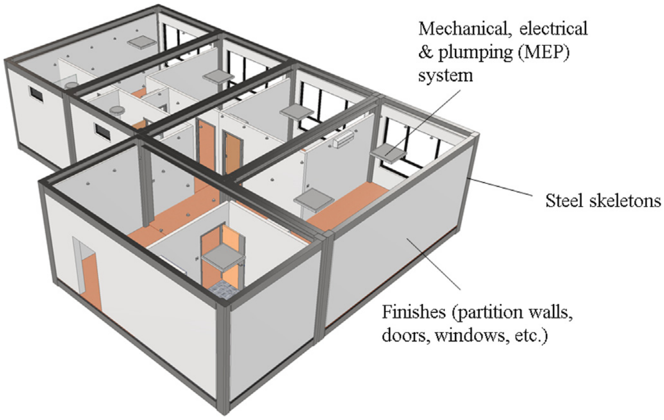


Fig. 1. Steel modules completed with MEP and finishes.

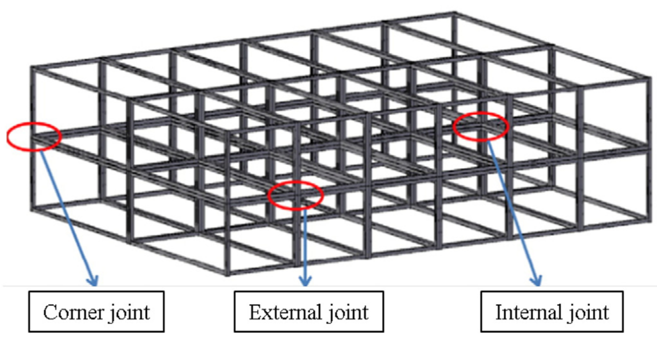


Fig. 2. Modules connected by (a) corner joint, (b) external joint, and (c) internal joint.

types: beam-beam and column-column connections [9]. Beam-beam connections were proposed to tie the floor beams of the upper modules to the ceiling beams of the lower modules [18–21]. In some of the designs, short segments like plug-in device, shear key or tenon were incorporated to enhance the shear force resistance and rotational stiffness between the upper and lower columns [22–25]. Nonetheless, for the modern modular construction, particularly for PPVC that comes with complete internal finishes, opening or access for bolting is not desirable [9]. For this reason, column-column connections were proposed to join the upper columns to lower columns using vertical rod, plugin bars, or pretension strands [9,26].

In some studies, novel inter-module connections were proposed, and simplified connection models were adopted in global analysis. In general, the design of the inter-module connections needs to consider vertical and horizontal connectivity. In a typical inter-module connection, the horizontal connectivity can be modelled by connecting a frame element between the adjacent columns. Some studies used spring elements to represent the stiffness properties of the inter-module connection [21,27]. Fathieh and Mercan modelled the horizontal connection as an element with bending and shear strengths 30% higher than the adjacent beam section [28]. On the other hand, the vertical connectivity can be modelled by extending the centerlines of columns beyond their intersections with the beams, before connecting it vertically with the lower columns [9,29]. Some studies suggested to simplify the vertical joint by simply assigning pin-ended constraint which is commonly used by practitioners to model the frame to predict its response behavior [18,21,28,30]. However, assuming pin-connected joints in modular buildings may not be conservative because it is uncertain that this assumption may lead to corresponding extremities in building sway behavior as would be expected in the conventional buildings where rigid joint assumptions are commonly adopted. Assuming pinned joints could also lead to false effective length of columns, overestimating the column capacities in an unbraced frame whereas it is a conservative assumption in a braced frame [31,32]. Furthermore, the joint which has some rigidity may attract forces and moments to its connecting components and these additional local stresses imposed on the joint are neglected. For this reason, the pin joint model may not be able to capture the true behaviour the actual inter-module joints. Therefore, it is important to understand the actual behavior of the joint and develop proper methods of modelling it in global analysis as it could affect the overall stability of the building.

In traditional buildings, the reinforced concrete slab is continuous throughout the entire floor and it is common practice to assign continuous rigid floor diaphragms to the entire slab in the global analysis. Likewise, current practice of modular construction involves in-situ grouting, rebars, plates or bolting to ensure the slabs or beams of adjacent modules are tied together to provide the diaphragm action [11,12,16,28]. By tying together all the slabs or edge beams of the modules, a continuous rigid diaphragm can be assigned at each floor level, like the conventional building. However, connecting the slabs of all the modules need more workers at the site and can slow down the speed of construction.

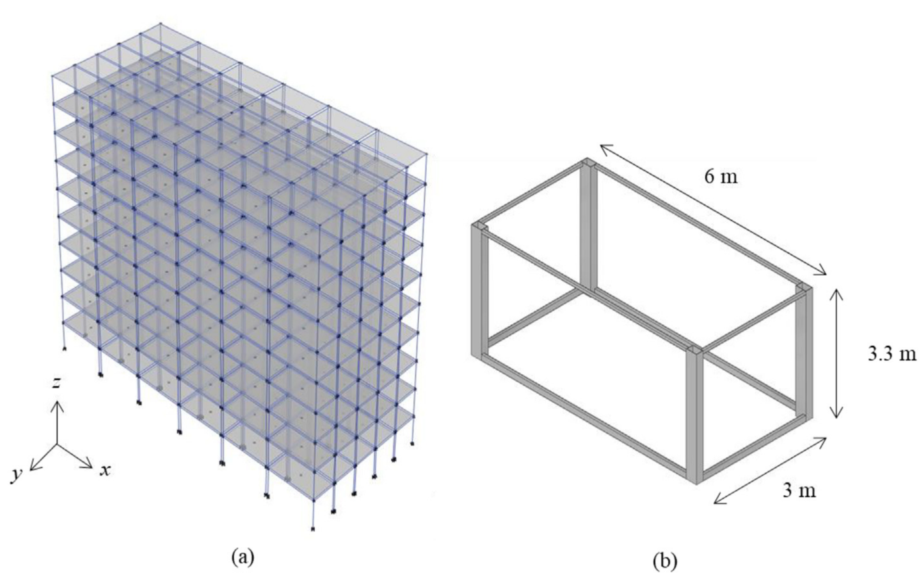


Fig. 3. (a) 10-storey unbraced modular building, and (b) single module.

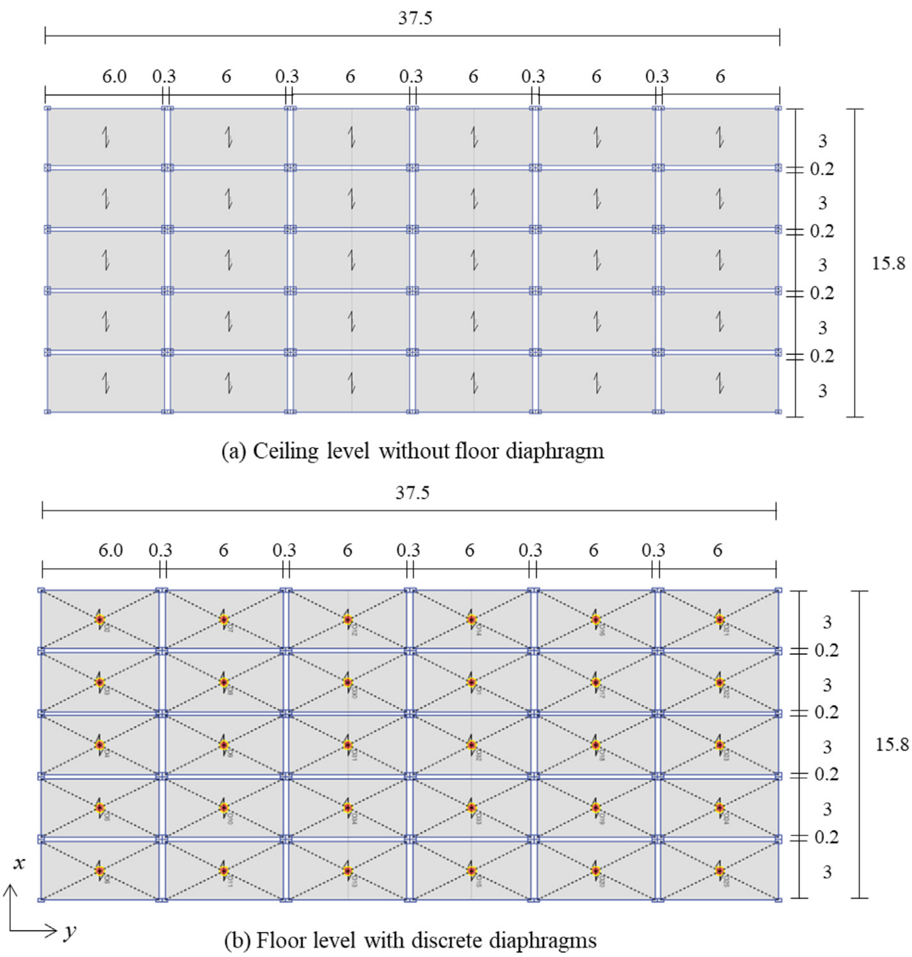


Fig. 4. Plan view of 10-storey unbraced modular building (a) ceiling level, and (b) floor level (all units in m).

If the modules are connected only at the corners, it is a more realistic to model the floor slab of each module as a discrete rigid diaphragm [9,11,28]. However, the effect of discontinuity of floor slab and the relative movements between the modules require further investigation. It is reported that an increase in diaphragm flexibility may result in increased inter-storey drift [12]. The horizontal load transfers through discrete floor diaphragms to the lateral force resisting system in the building has not been fully studied.

The main purpose of the present research was to study the effects of connection modelling and floor diaphragm discontinuity on the global sway behaviour of high-rise modular buildings with different lateral force resisting systems. Firstly, simplified joint models with pin or rigid fixities suggested by several researchers, were studied and compared with the proposed spring model to ensure its accuracy in capturing actual joint behaviour. Detailed studies were conducted on a 10-storey unbraced modular building frame, followed by adding steel bracing in the modular units to improve their lateral resistance. In addition, a 40-storey steel modular building with reinforced concrete core walls serving as lateral load resisting system was analyzed. The analysis model included the semi-rigid effect of inter-module connections on the lateral stability and side sway behavior of the modular building.

2. Buildings designed for modular construction

A 10-storey residential building, as shown in Fig. 3(a), was designed based on prefabricated prefinished volumetric construction with free-standing volumetric modules (complete with columns, beams, walls, floor and ceiling slabs) manufacturing in a factory, followed by on-site installation. The dimensions of the module are 6 m × 3 m × 3.3 m (*L* x

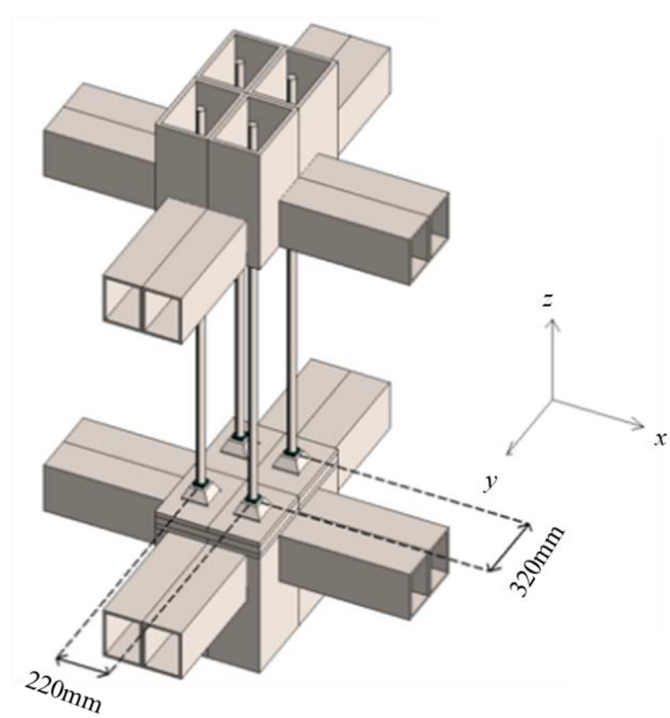


Fig. 5. Exploded 3D view of an internal inter-module joint connecting eight modules (4 top and 4 bottom).

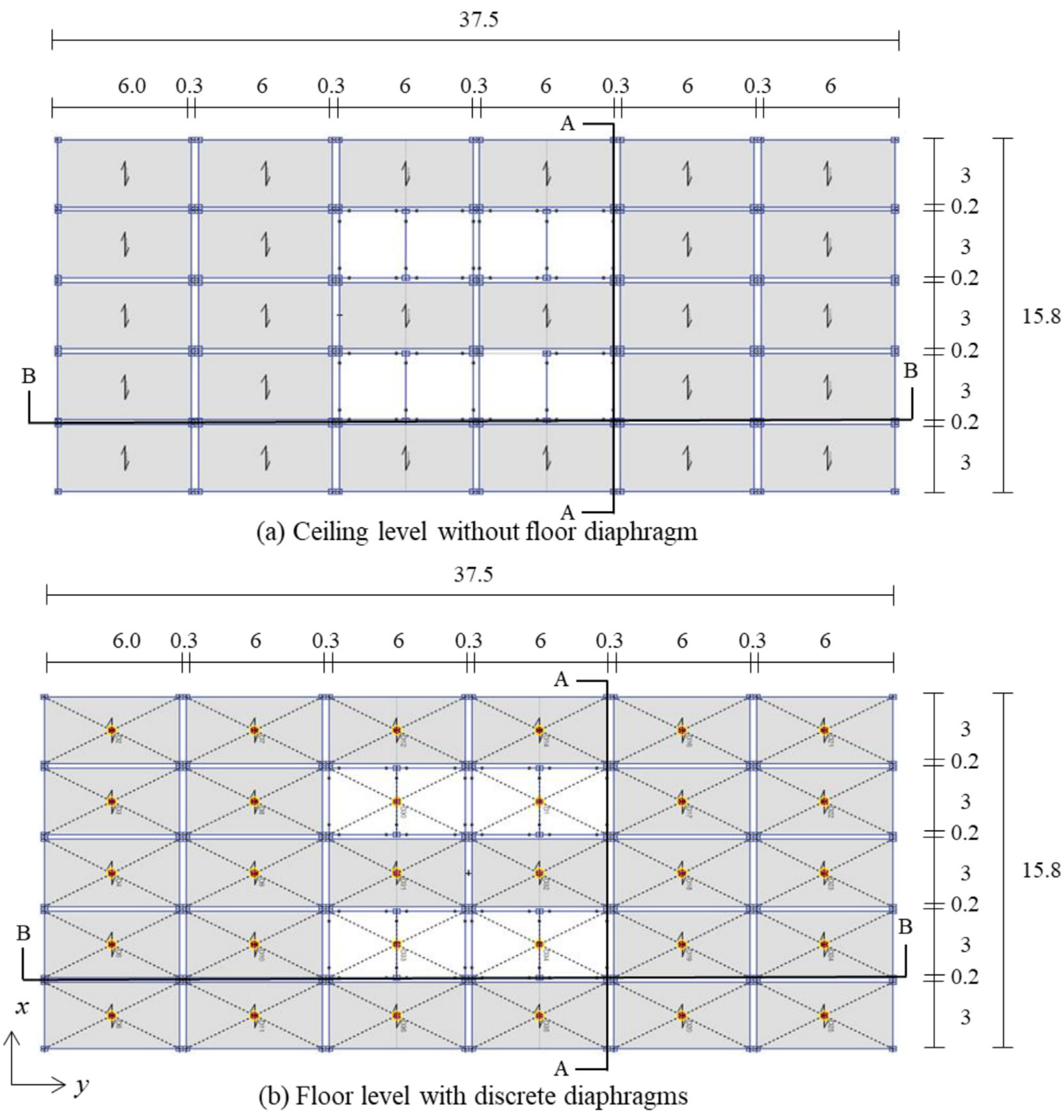


Fig. 6. Plan view of 10-storey modular building with central braced modules (a) ceiling level, and (b) floor level (all units in m).

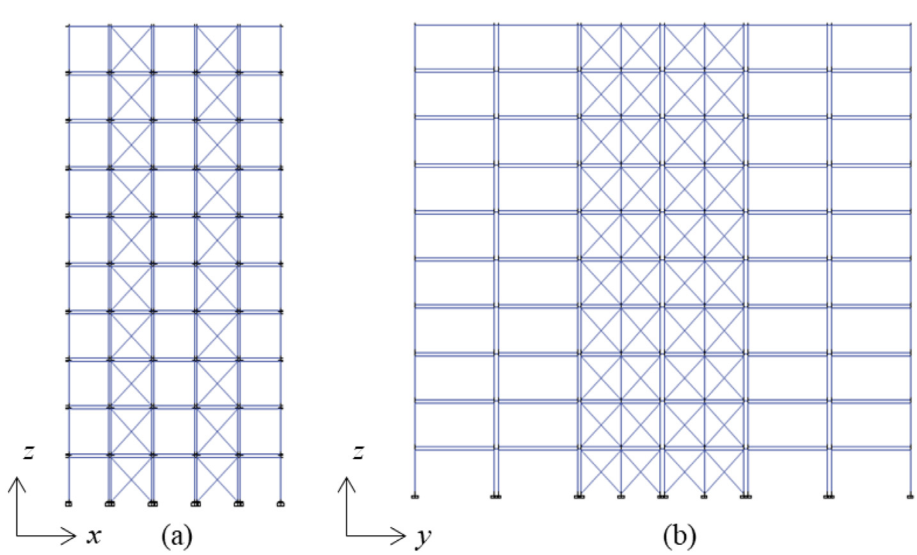
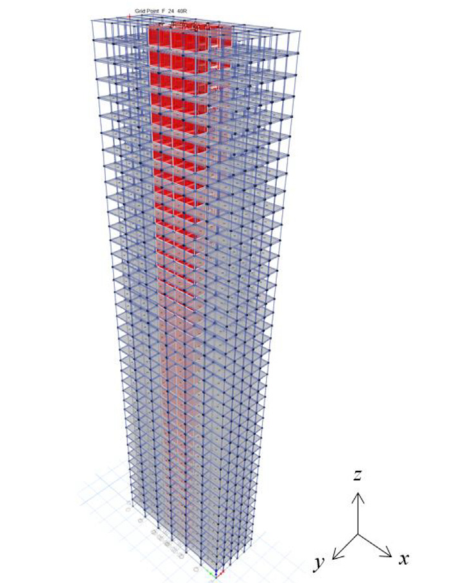


Fig. 7. Elevation view of 10-storey residential modular building with central braced modules (a) A - A and (b) B - B (all units in m).



Total floor area (m^2)	15.8 x 37.5 = 593
Module/Total floor area (%)	82

Fig. 8. 3D view of 40-storey modular building [9].

B x H) as shown in Fig. 3(b). The building height is 35 m, and all the columns are rigidly fixed to the ground. Each module has a ceiling and a floor slab as shown in Fig. 4(a) and (b).

Each module was modelled with 220 mm and 320 mm gaps in the x- and y- axes to account for the centre-line offset of columns and a small gap (e.g. 20 mm) between two adjacent columns as depicted in Fig. 5. To increase the lateral resistance of the unbraced modular building, steel bracings were added to the modules located at the center of the building as shown in Figs. 6 and 7.

Thereafter, various connection models as well as rigidity of floor diaphragm on the lateral behaviour of the frame were investigated by studying a 40-storey residential modular building which was braced by a reinforced concrete core. Although similar building as shown in Figs. 8 and 9 was used in the previous study [12] to discuss the global modelling of modular building, the present study investigated the connection models as well as the side sway behvaiour of the modular building under lateral loads. The modules were arranged around the centre core walls which provided lateral stability to the building as shown in Figs. 8 and 9. The structural modular units were designed to resist the gravity loads whereas the central core walls resisted the lateral loads. Due to the high lateral stiffness of the core walls, about 95% of the base shear was resisted by the core walls when the columns and core walls were rigidly connected to the foundation.

Another important factor that affects the side-sway stability of the modular buildings is the connection between the modules and the core walls. Fig. 10 shows a typical connection design between core

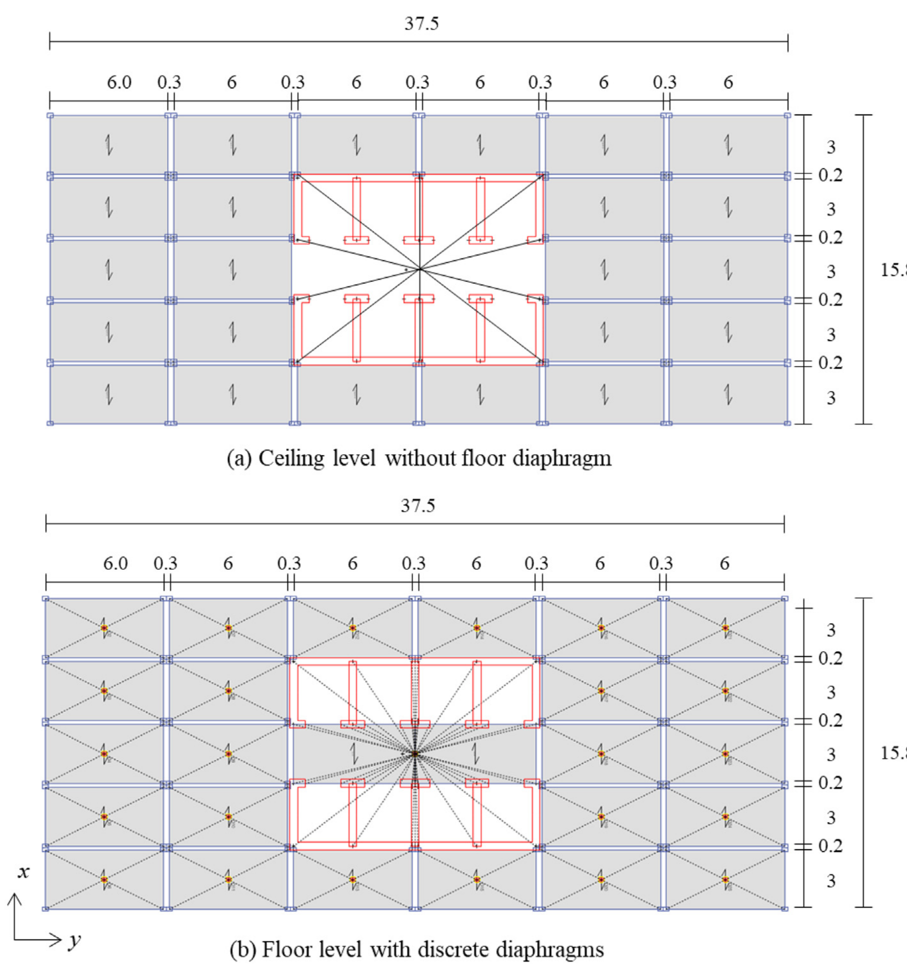


Fig. 9. Plan view of 40-storey modular building with reinforce concrete (RC) core (a) ceiling level, and (b) floor level (all units in m) [9].

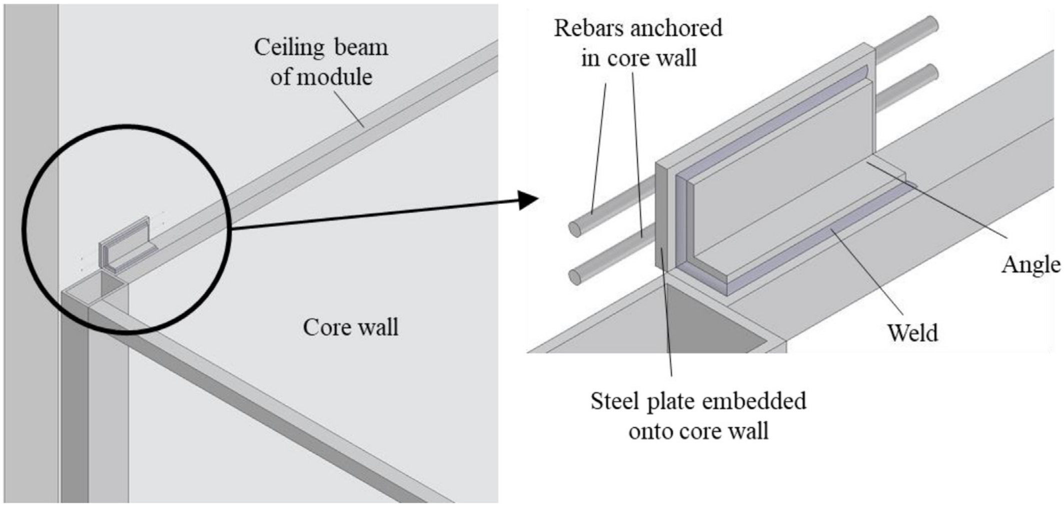


Fig. 10. Example of fix-ended connection between core wall and modules.

wall and modules. An angle section attached to the ceiling beam of the module is welded to the steel plate embedded in the core wall. This connection enables the transfer of lateral loads acting on the modules to the concrete core walls. Hence, it is common to model this horizontal connection as fix-ended short frame element in numerical analysis as shown in Fig. 10.

To promote standardization of the members and connection details, same column size was adopted throughout the height of the building. Same rectangular steel section with difference wall thickness was used for the columns for the ease of connecting modules in the vertical direction. Table 1 shows the beam and column section sizes used in the analysis model. The super-imposed dead and live loads are given in Table 2. Second order elastic analyses were carried out using global frame imperfections. However, member imperfection and curvature were not captured in the analysis and thus member stability checks were carried out separately. The global sway behaviour of the modular buildings was studied from the lateral force-displacement curves. Only the lateral displacement under wind load in the *X* direction is presented in this paper because it is more critical than that of the wind load acting in the *Y* direction.

Fig. 11 shows a typical column to column connection that is used to connect the upper module to the lower module in a typical steel modular building [9]. The modules are fastened vertically by a threaded steel rod, which is secured by a locking nut at each floor level. This ensures vertical continuity of the columns and allows the transfer of moments and shear forces to the foundation. Shear forces are transferred between the vertically stacked modules via shear key bearing against a base plate welded to the column, as shown in Fig. 11. Compressive forces are transferred through the connection via direct bearing between the top and bottom columns. Tension forces (if any) are resisted by the vertical rod. The modules are tied horizontal by a steel tie plate connecting the shear keys of the adjacent columns. Shear and axial forces are transferred between the modules via bearing and shearing of the shear keys against the tie plate. The details of inter-module connection modelling (i.e. simplified and spring models) are discussed in Section 3.

The effect of floor diaphragms (continuous versus discrete diaphragm) was studied by comparing the load displacement response of the building with the more exact analysis that modelled the floor slab using shell elements to simulate more accurately the in-plane stiffness as depicted in Fig. 12. As the ceiling of the module was made from 2 mm thick engineered cementitious composite (ECC) board supported by steel batons, it did not have enough in-plane stiffness to act as a floor diaphragm to transfer lateral load. Hence its contribution was not included in the frame modelling.

For the cases where the floor slabs were modelled using discrete rigid diaphragms or shell elements as shown in Fig. 12(b) and (c), wind load could not be assigned at the centroid of the floor slab. It was then assigned on the external cladding wall modelled using arbitrary membrane elements with no material property as depicted in Fig. 13. Horizontal loads including wind and equivalent horizontal loads were applied on each cladding panel. Alternatively, semi-rigid diaphragms were assigned to all the joints at each floor level. Comparing to rigid diaphragms which have infinite in-plane stiffness properties, semi-rigid diaphragms simulate actual in-plane stiffness of the slabs and the flexibility of the inter-module connections can also be modelled directly [33]. Using this method, lateral loads such as wind load can be assigned conveniently to the centroid of each diaphragm. The purpose of these comparative studies was to find out if the simplified diaphragm models can be used as compared to the more exact shell element

Table 1
List of member material and sizes used in analysis model.

Member	Material grade	Section size
Ceiling beam	S355 steel	SHS 80 × 80 × 5
Floor beam	S355 steel	RHS 160 × 80 × 6.3
Column 1 (Level 1–10)	S355 steel	RHS 300 × 200 × 16
Column 2 (Level 11–20)	S355 steel	RHS 300 × 200 × 12.5
Column 3 (Level 21–30)	S355 steel	RHS 300 × 200 × 8
Column 4 (Level 31–40)	S355 steel	RHS 300 × 200 × 6.3
Concrete Floor slab	Concrete C35/45	130 mm thick
Ceiling deck	Engineering cementitious composite with steel baton	2 mm thick
Steel plate (*x*-axis)	S460	320 mm×20 mm
Steel plate (*y*-axis)	S460	220 mm×20 mm

Table 2
Super-imposed dead and live loads in the analysis model.

Structural component	Load type	Load
Floor slab	Super-imposed dead load	1.2 kN/m^2
	Live load	1.5 kN/m^2
Ceiling level	Super-imposed dead load	1.0 kN/m^2
	Live load	0.75 kN/m^2
Corridor slab	Super-imposed dead load	1.2 kN/m^2
	Live load	3.0 kN/m^2
Floor beam	Super-imposed dead load	4.2 kN/m

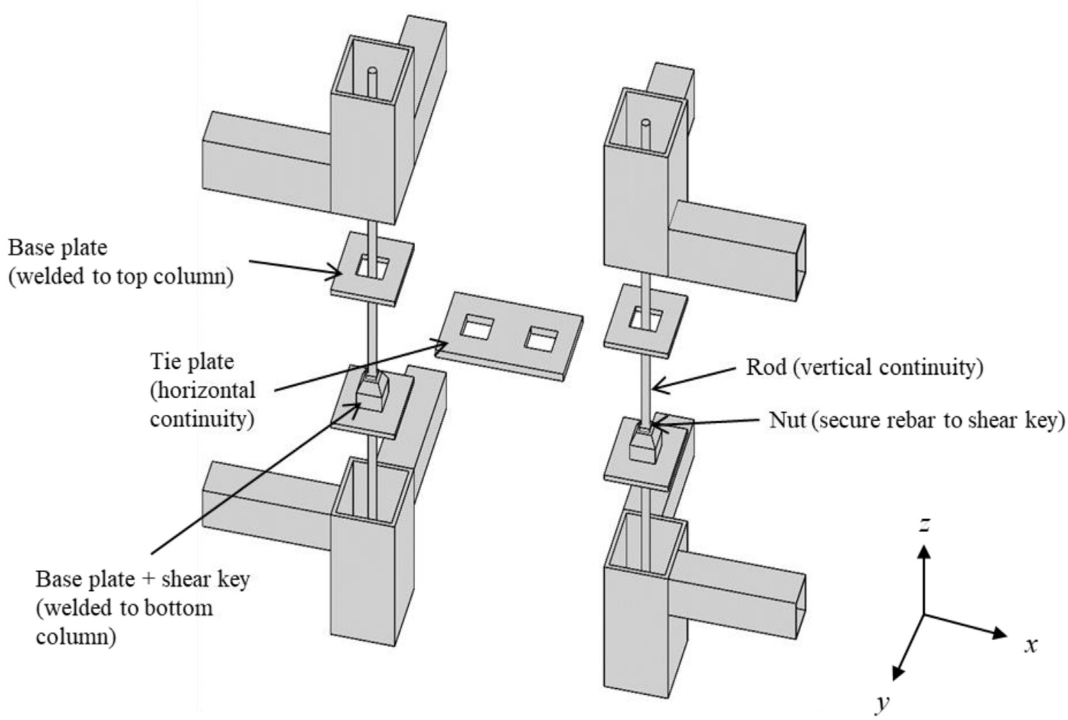


Fig. 11. Exploded view of exterior inter-module connection with labels of its essential mechanisms [9].

approach in modelling the floor slab so that the most appropriate and convenient method can be adopted for practical design.

3. Inter-module connection model

In this section, the details of inter-module connection modelling are discussed. The stiffness parameters of a joint's constituent mechanism are found separately before they are combined to form a mechanical model representative of the actual joint [34]. Referring to the typical inter-module connection as shown in Fig. 11, it consists of vertical and horizontal connections. The columns from the upper and lower modules are not continuous. The vertical continuity relies solely on the vertical rod. Therefore, the columns are designed to resist the compressive load, while the vertical rod is to resist tension and to prevent the modules from separating vertically under high lateral forces. The shear force between upper and lower modules is resisted by the bearing action of the top plate from the upper column on the shear key. The horizontal force is transferred from one module to the adjacent one by the horizontal tie plate bearing against the shear key. Tables 3 and 4 summarize the degree of freedoms (DOFs) for the vertical and horizontal connections.

Fig. 14 shows the simplified joint models with the assumptions of pin and rigid fixities used in global modelling of a modular building. The simplified joint model was proposed and commonly used by practitioners due to the ease of modelling a three-dimensional (3D) framework. In this model, the inter-module vertical connection was modelled by extending the centerline dimension of column beyond the intersection point with the floor beam, before connecting it vertically with the lower column. For simplicity the vertical connection was modelled as a pin joint because the upper and lower modules were connected only by the tension rod. To do so, the bottom part of the upper column was pin-connected to the lower column as depicted in Fig. 14(c). Nonetheless, as mentioned in Section 1, it is uncertain whether the pinned vertical connection is a conservative assumption in designing the modular buildings. Another extreme fixity in vertical connection design is to rigidly connect the upper and lower columns as shown in Fig. 14(b).

In the horizontal connection, a tie plate was used to connect the adjacent modules, but the effective shear and axial lengths were simplified to span between the centers of adjacent shear keys instead of between their inner edges as shown in Fig. 15. It was modelled by connecting a frame element with tie plate properties between the midpoints of the adjacent columns. The tie plate was assumed to be rigidly connected to the adjacent columns as shown in Fig. 14(c) because the clamping forces imposed by the columns were assumed to restrict relative rotation of the tie plate.

The current simplified joint models shown in Figs. 14(b) and (c) assumed that the horizontal tie plate was rigidly connected to the lower columns. This resulted in increased joint stiffness from the columns of the lower modules and might not be a conservative assumption because the tie plate was simply placed on top of the lower columns and held in place by the shear keys as shown in Fig. 11. Likewise, modelling the horizontal tie plate with pin end condition might cause the plate to act as an axial member, neglecting its in-plane shear stiffness. Fig. 14 (d) shows that the horizontal tie plate was only pinned at the out-of-plane axis. Hence, the moment could not be transferred between the columns and the tie plate.

The simplified joint models as depicted in Fig. 14 utilized the column properties to resist both compressive and tensile loads in the vertical connection. This assumption may not conservative since the tensile capacity of the column is contributed by the vertical rod which is significantly lower than that of the column. To investigate the feasibility of using simplified joint models, a translational spring model was proposed to simulate the actual behaviour of the inter-module connection, as shown in Fig. 11 and its mechanical behaviour was compared with those of the simplified joint models. The stiffness parameters of the vertical and horizontal connections with DOFs elaborated in Tables 3 and 4 were determined using analytical methods for the spring model. The vertical and horizontal connections were modelled using link elements as shown in Fig. 16. The stiffness parameters obtained were input into the link elements in the analysis model. The link elements were assigned with user-specified force-deformation relationships for each of its six DOFs.

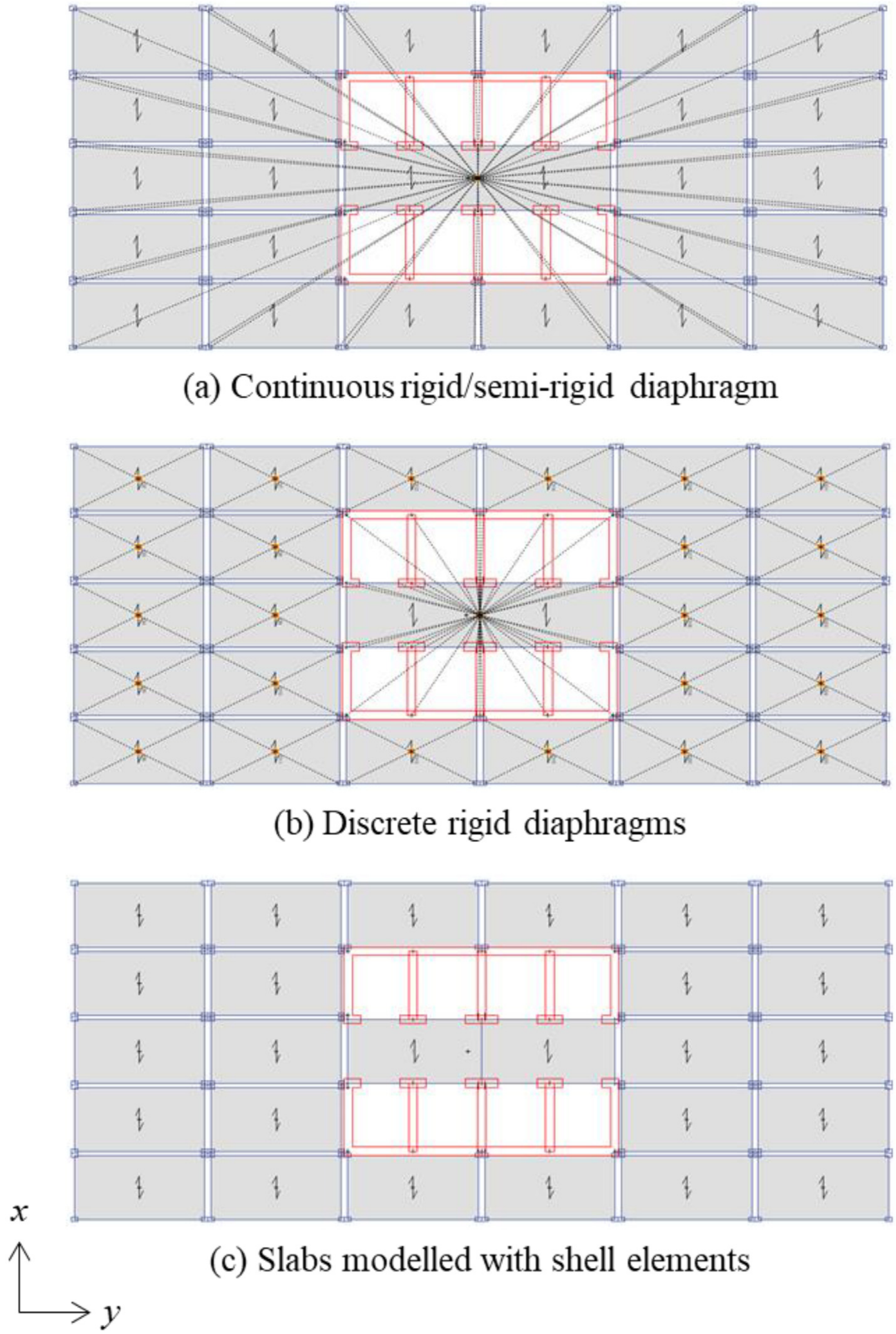


Fig. 12. Floor slabs with different modelling methods.

Referring to the vertical connection with DOFs elaborated in Table 3, the compressive load is resisted by the column whereas tensile load is taken by the rod that ties the columns vertically. The connection of the rod to the module was detailed in such a way that only one-storey height of the rod is under tension when a joint is subjected to tensile loads. The axial compressive and tensile stiffness of vertical mechanism were calculated analytically using Eq. 1. Rotational stiffness was negligible because it was contributed by the axial elongation of the tension rod. Hence it was conservatively assumed to be zero in the proposed spring model.

$$k_{axial} = \frac{AE}{L} \tag{1}$$

where

A = Cross sectional area; E = Young's Modulus of member L = Axial length.

The resistance against lateral movement between the top and bottom modules come from the column base plate bearing against the shear key. The shear force was assumed to be distributed evenly across the width of the base plate towards the shear key. The vertical shear stiffness of the vertical mechanism can be idealized to be equivalent to the shear stiffness of the portions of the base plate under shear force (shaded in.

Table 5). The shear stiffness of the base plate was modelled as:

$$k_{shear} = \frac{AG}{l} \tag{2}$$

where

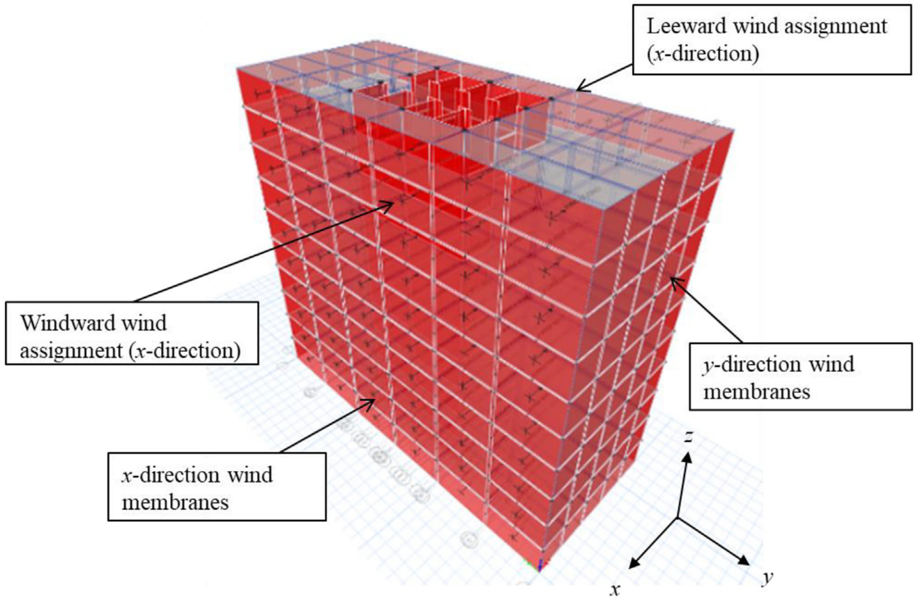


Fig. 13. Illustration of wind load assignment on arbitrary cladding in the analysis model.

A = Cross-sectional area; G = Shear Modulus of material; l = Shear length.

Table 5 summarizes the inputs for vertical connection of proposed joint model using Eqs. 1 and 2.

On the other hand, the DOFs for the horizontal connection as demonstrated in Table 4 are mainly resisted by the components such as steel plate and shear key through axial and shear forces. Thus, the axial and shear stiffness parameters were also determined using an

Table 3
Illustration of DOFs for vertical connection.

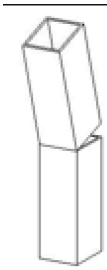
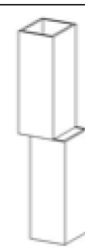
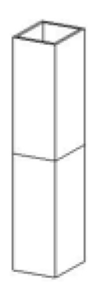
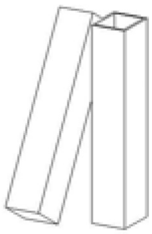
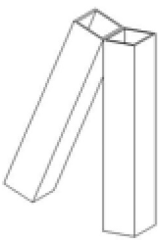
Rotation DOFs	Remarks	Translation DOFs	Remarks
 *x-x* rotation (major axis)	• Affects lateral movement between modules	 *x*-direction shear	• Affects lateral movement between modules
 *y-y* rotation (minor axis)	• Affects lateral movement between modules	 *y*-direction shear	• Affects lateral movement between modules
 *z-z* torsion	• Unlikely to occur due to volumetric nature of the modules • Even if it occurs, does not affect lateral movement between modules	 *z*-direction axial	• Transfers vertical loads between columns

Table 4
Illustration of DOFs for horizontal connection.

Rotation DOF	Remarks	Translation DOFs	Remarks
 *x*-*x* rotation	• Likely to occur due to construction/manufacturing tolerance, accidental loads, differential settlement, etc.	 *x*-direction axial	• Likely to occur due to transfer of lateral loads between adjacent modules
 *y*-*y* rotation	• Likely to occur due to construction/manufacturing tolerance, accidental loads, differential settlement, etc.	 *y*-direction shear	• Likely to occur due to transfer of lateral loads between adjacent modules
 *z*-*z* rotation	• Likely to occur due to construction/manufacturing tolerance, accidental loads, differential settlement, etc.	 *z*-direction shear	• Likely to occur due to transfer of lateral loads between adjacent modules

analytical method. As the dimensions of the horizontal connection differ in the two plane directions as demonstrated in Fig. 5, there are two sets of stiffness parameters for the horizontal connection. Similar to vertical connection, both the axial and shear stiffness parameters of horizontal connection were determined using Eqs. 1 and 2 and are tabulated in Table 6. In the event of differential settlement occurring between the adjacent columns, the clamped portions of the tie plate are restrained from the vertical shearing action due to the vertical bearing force being exerted by the columns on to the plate. Therefore, the portion of the horizontal tie plate that experiences vertical shear can be idealized to be equivalent to the 20 mm buffer zone between the columns. As discussed for the simplified model shown in Fig. 14(d), the horizontal tie plate is simply placed between the two columns and is held in place by the shear key. Hence, it was assigned as pinned at out-of-plane axis while fixing the in-plane rotational restraint.

A summary of stiffness parameters for the vertical and horizontal connections of the spring model and their comparisons with those of the simplified joint model are tabulated in Tables 7 and 8 respectively. The differences between the simplified and spring models for the vertical connection are the magnitudes of axial tensile stiffness and shear stiffness in the two planar directions. The magnitudes of shear stiffness of the simplified model are contributed by the columns while those of the spring model are from the base plate and shear key. For the horizontal connection, the axial and shear stiffness parameters differ between the simplified and the proposed spring models. Their difference lies solely in the difference between axial and shear lengths adopted in the calculations. The simplified model derives its axial and shear stiffness parameters by using the tie plate length between the centers of two adjacent shear keys, while the spring model adopts the exact length between the inner-edges of the shear keys as illustrated in Fig. 15. The horizontal connection for the proposed spring model is stiffer than that of the simplified model. Using the proposed spring model, the feasibility of the simplified joint models with assumption of pin and rigid fixities to capture the actual behavior of inter-module joint was analyzed and compared based on their load displacement responses. Table 9 summarizes the various joint models investigated in this paper.

4. Modular framing system

To investigate the effects of different joint models tabulated in Table 9, the lateral load resistance of a 10-storey unbraced modular building was firstly examined. Bracings were added to some of the modules to explore the feasibility of having a modular braced frame in modular structures. Thereafter, a 40-storey modular building with a reinforced concrete core wall was used to study the effects of joint models as well as the various floor diaphragm assignments. In order to explore the feasibility of using the idealized joint model in modelling, the spring model (i.e., case 4 in Table 9) was studied and compared with the simplified joint models that consist of different end restraints (i.e., cases 1, 2, and 3 in Table 9).

4.1. Unbraced modular frame system

To understand the effects of joint modelling on the global behaviour of a steel modular framing system, a 10-storey unbraced buildings as shown in Figs. 3 and 4 with different joint models were examined. All the modules were modelled using discrete rigid diaphragm in this section. Fig. 17 shows the lateral force-displacement curve of the modular

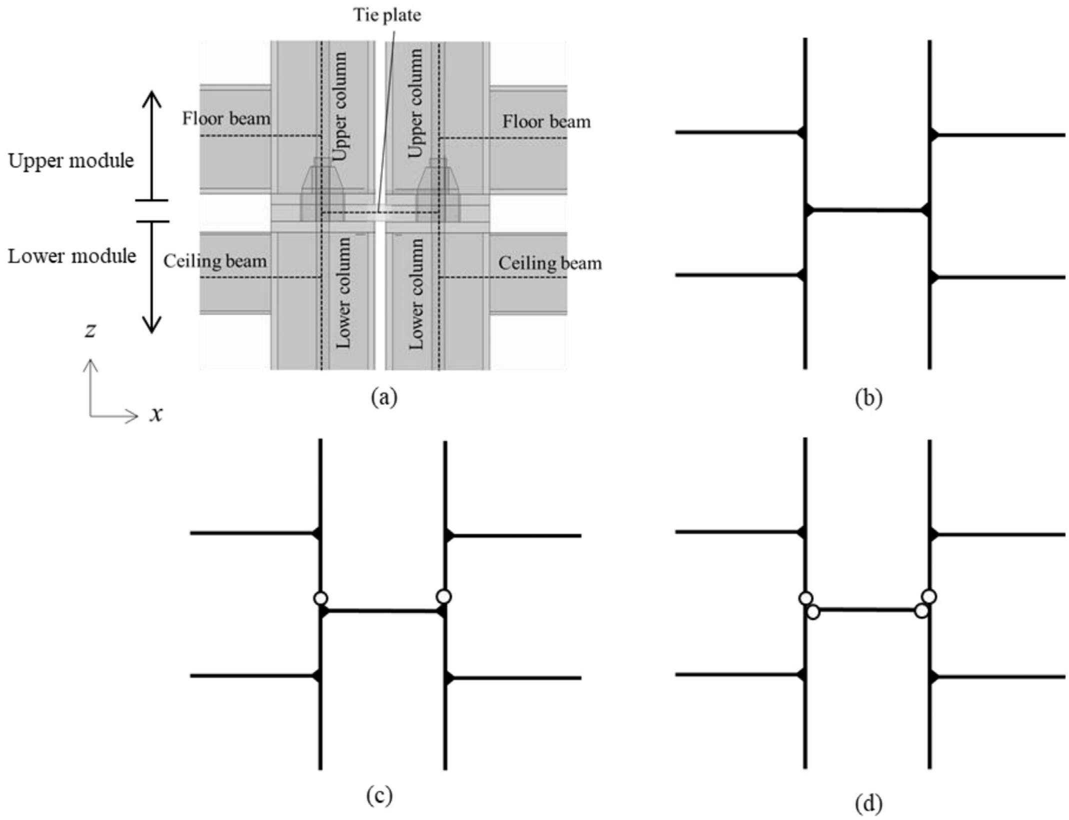


Fig. 14. Inter-module connection (a) schematic view, with its simplified joint models showing (b) fix-ended vertical and horizontal connections, (c) pin-ended vertical connection and fix-ended horizontal connection, and (d) pin-ended vertical connection while having pin-ended horizontal connection (releasing the out-of-plane rotational restraint and fixing in-plane rotational restraint).

building by varying the inter-module joint models when subject to wind load in the *X* direction. Pin-ended vertical connections (Case 2) is observed to have a slight reduction in lateral stiffness of about 15%, as compared to the fix-ended vertical connection (Case 1). This is caused by similar bending behaviour at the inter-module joint for Cases 1 and 2 whereby zero bending moment occurs between the upper and lower modules as demonstrated in Fig. 18. On the other hand, by releasing the out-of-plane rotational restraint of the horizontal tie plate while fixing its in-plane rotational restraints (Case 3), its lateral stiffness drops about 15% as compared to the case with horizontal tie plate modelled as fix-ended (Case 2). This shows that for Cases 1 and 2, which are commonly used in industrial practice, are not conservative in capturing the global behaviour of a modular building. As shown in Fig. 19(a), to model a fix end restraint of the tie plate, the column from the upper module is pin-ended to the tie plate, while the column from the lower module is rigidly connected to the tie plate. This results in increased stiffness of the column from the lower module, overestimating its stiffness in the design. By releasing the out-of-plane rotational restraint while fixing the in-plane rotational restraint, the lateral deformed shape of the joint model is more realistic as shown in Fig. 19 (b), because the horizontal tie plate is just simply placed on top of the column.

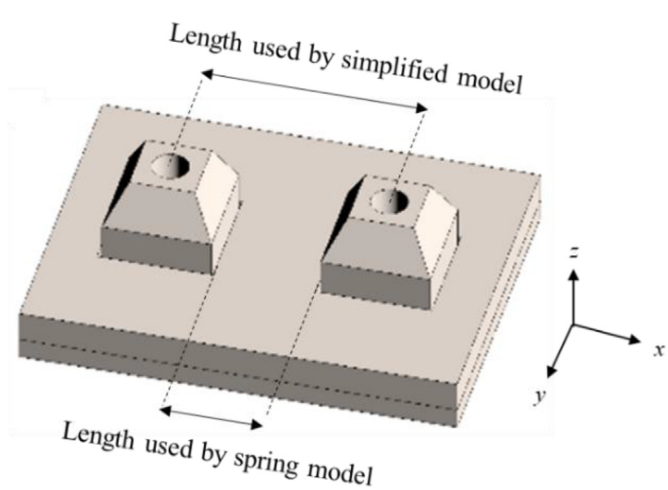


Fig. 15. Lengths for calculating the axial and shear stiffness of the connecting element between the two adjacent columns.

Lastly, the spring model (Case 4) shows slight reduction in lateral stiffness when compared to Case 3. This is because the spring model explicitly models the axial stiffness of the tension rod, whereas the simplified joint model utilizes the column properties which have relatively higher tensile stiffness. Under lateral force, significant amount of tensile force may be induced in the columns especially for low- and mid-rise unbraced buildings whereby there is no lateral force resisting system to resist the lateral loads and hence the tension rods in the columns have to resist the lateral loads. Therefore, using simplified joint in the global modelling for modular buildings tends to overestimate the tensile stiffness of the vertical connection for an unbraced modular framework.

4.2. Steel braced modular frame

In this section, diagonal steel braces were added into the modules located at the central part of the building as shown in Figs. 6 and 7 to enhance the lateral stiffness of the modular building against side sway. An independence lateral force resisting systems such as concrete core wall is not needed as the construction of which tends to slow down the speed of construction.

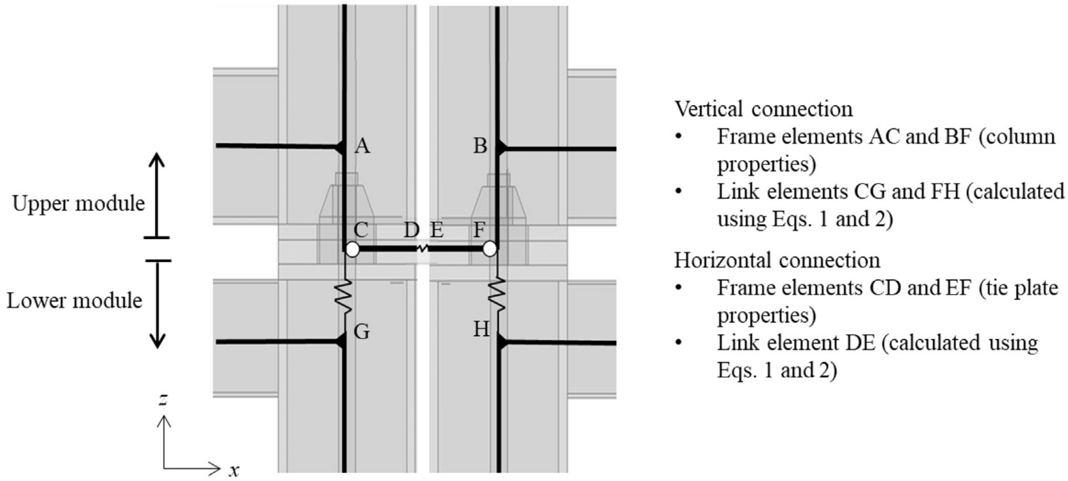


Fig. 16. Proposed translational spring model.

Fig. 20 shows the lateral force-displacement curve of the 10-storey modular building with steel bracings while varying the inter-module joint models under wind load X. As can be seen, the lateral resistance of all the cases with steel bracings increases significantly as compared to those of unbraced system except for the case with the spring model (Case 4). Case 4 is observed to have a lateral stiffness 70% lower than the case with the horizontal tie plate modelled as freely rotate against the out-of-plane rotation and fixed against the in-plane rotation (Case 3). This is because by adding steel bracings to the modules, the frame becomes stiffer. Hence, the applied lateral load tends to amplify the toppling effect of the stacked up modules as demonstrated by the deformed shape of the building in Fig. 21(b). The tension rod in the columns plays an important role in controlling the sides way behavior of the braced modular building.

4.3. Incorporating reinforced concrete braced core

For high-rise buildings, it is common to utilize reinforced concrete braced core to provide lateral stability to the building. A 40-storey modular building with structural schemes as shown in Figs. 8 and 9 was used to investigate the lateral resistance of high-rise modular buildings with different joint models. This was done simultaneously with the effects of diaphragm action to determine the feasibility of connecting adjacent units only at the corner joints rather than along the adjacent beams or slabs. Connecting the modules at four corners enhances the efficiency and productivity of a modular building as compared to a conventional building, which would normally have a cast in-situ monolithic slab at every floor level. Nonetheless, discontinuity of floor slabs in a modular building may affect the lateral load transfer mechanism of the structure.

Table 5
Input parameters for vertical connection of translational spring model using Eqs. 1 and 2.

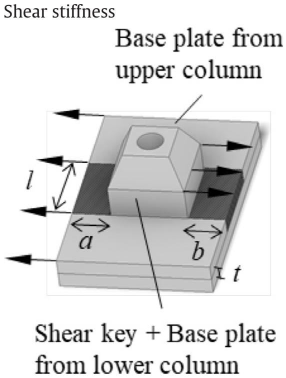
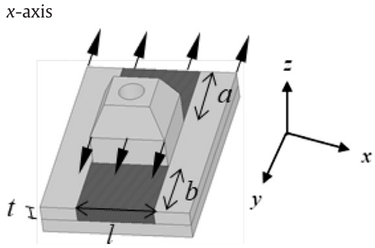
Axial stiffness	Cross sectional area *A* (mm^2)	Axial length *L* (mm)
Compression	Column	Vertical link length (Refer to Fig. 16)
Tension	Rod	Height of one storey
Shear stiffness	Cross-sectional area *A* (mm^2)	Shear length *l* (mm)
 *x*-axis  *y*-axis	$(a + b) \times t$	*l*

Table 6
Input parameters for horizontal connection of translational spring model using Eqs. 1 and 2.

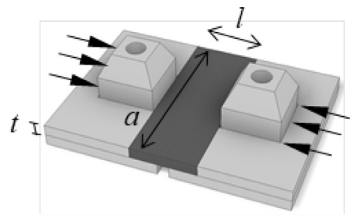
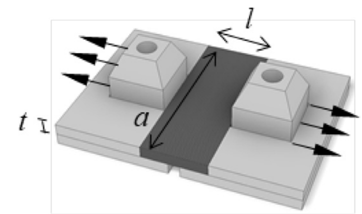
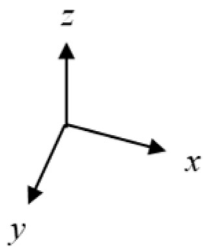
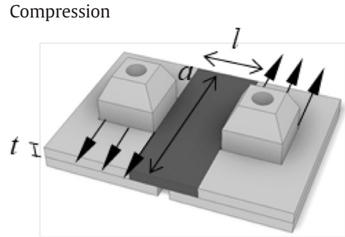
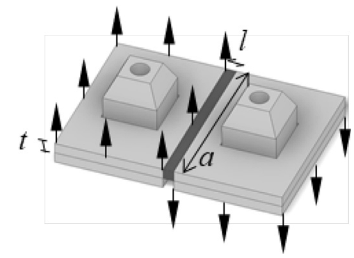
Stiffness mechanism	DOF notation	
Axial	 Compression	 Tension
 Shear	 Horizontal shear	 Vertical shear
Cross sectional area A (mm^2)	a x t	
Length L (mm)	l	

Table 7
Summary of stiffness parameters for vertical connection of simplified and translational spring models.

Stiffness parameter	Simplified	Spring	Difference (%)
Axial compression column 1[a] (kN/m)	24.7×10^3	24.7×10^3	0
Axial compression column 2[a] (kN/m)	19.7×10^3	19.7×10^3	0
Axial compression column 3[a] (kN/m)	12.9×10^3	12.9×10^3	0
Axial compression column 4[a] (kN/m)	10.2×10^3	10.2×10^3	0
Axial tension (kN/m)	Similar to compression	0.04×10^3	−52,600
Shear – x axis (kN/m)	4.9×10^3	1.5×10^3	−230
Shear - y axis (kN/m)	3.2×10^3	3.0×10^3	−8

[a] For description, refer to Table 1.

Table 8
Summary of stiffness parameters for horizontal connection of simplified and translational spring models.

Stiffness parameter	DOF notation	Simplified	Spring	Difference (%)
Axial (kN/m)	x-axis	6.3×10^3	11.0×10^3	43
	y-axis	2.8×10^3	3.9×10^3	29
Horizontal shear (kN/m)	x-axis	2.4×10^3	4.3×10^3	43
	y-axis	1.1×10^3	1.5×10^3	29
Vertical shear (kN/m)	x-axis	2.4×10^3	24.3×10^3	90
	y-axis	1.1×10^3	16.2×10^3	93

Table 9
Summary of various joint models used in building models.

Case	Model	Rotational restraint	
		Horizontal connection	Vertical connection
1	Simplified	Fix	Fix
2	Simplified	Fix	Pin
3	Simplified	Pin (in-plane fixed)	Pin
4	Translational spring	Pin (in-plane fixed)	Pin

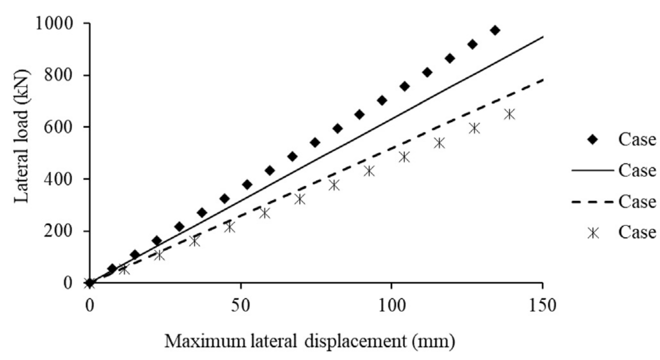


Fig. 17. Lateral force-displacement curves of 10-storey unbraced modular buildings with varying inter-module joint models under wind load in X-direction (Refer to Table 9 for the joint models).

For this reason, both discrete and continuous types of diaphragm assignments were studied and the lateral responses were compared with that of a model using shell elements as shown in Fig. 12.

The effect of different joint models on the lateral resistance of high-rise modular buildings with RC core walls was discussed and modelled using discrete rigid diaphragms. The results in Fig. 23 show that despite the differences in joint models, the lateral resistance of the modular buildings with RC core walls show similar behaviour. This suggests that the global sway behaviour of high-rise modular buildings is not sensitive to the variation of inter-module connection models due to the existence of stiff core walls which dominates the lateral stiffness of the building. Therefore, the difference in tensile stiffness between the simplified and spring models is not critical in influencing the sway behavior of the building. The columns in the modules are subject to compression force because up to 95% of the lateral force is resisted by the RC

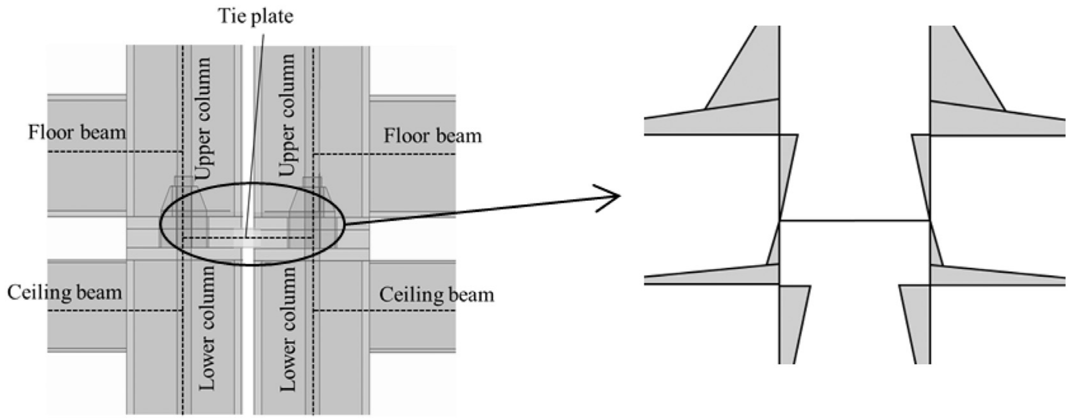


Fig. 18. Similar bending moment in the link element for joint model with fix-connection (i.e. Case 1) and pin-ended connection (i.e. Case 2) between upper and lower modules while the tie plate is fixed at both ends.

core wall, whereas the loads acting on the modules are dominated by the gravity action.

Some of modules in the building are not directly connected to the core walls. For such cases, the horizontal tie plate becomes much more critical in resisting the lateral loads especially when the modules are tied at the four corners by joints. The horizontal tie plate must have sufficient thickness to prevent swing-out effect, causing excessive deformation between the units that are not directly connected to the RC core walls as shown in Fig. 24(a). This scenario can also be caused by the modelling of the tie plate with pinned end restraints in both out-of-plane and in-plane rotational axes. Eliminating the in-plane stiffness of the tie plate causes the tie plate to be flexible and act as an axial member. Therefore, the in-plane shear stiffness of the tie plate is essential in resisting the lateral load and to ensure there is no excessive sliding between the adjacent modules under the effect of lateral load as depicted in Fig. 24(b).

The connection model in Case 4 (Table 9) was chosen to study the effect of different diaphragm assignments on the module slabs such as discrete diaphragms, continuous rigid diaphragm, and sab modelled using shell elements as shown in Fig. 12. Fig. 25 shows that the lateral load resistance of the modular building with discrete and continuous rigid diaphragms is similar to the frame with floor slabs modelled using shell elements. This is because of the relatively short effective length of the tie plate which provides high flexural and shear rigidity to resist shear movement between the adjacent modules. Moreover, the slab of each module is stiff enough to behave like a rigid body. It

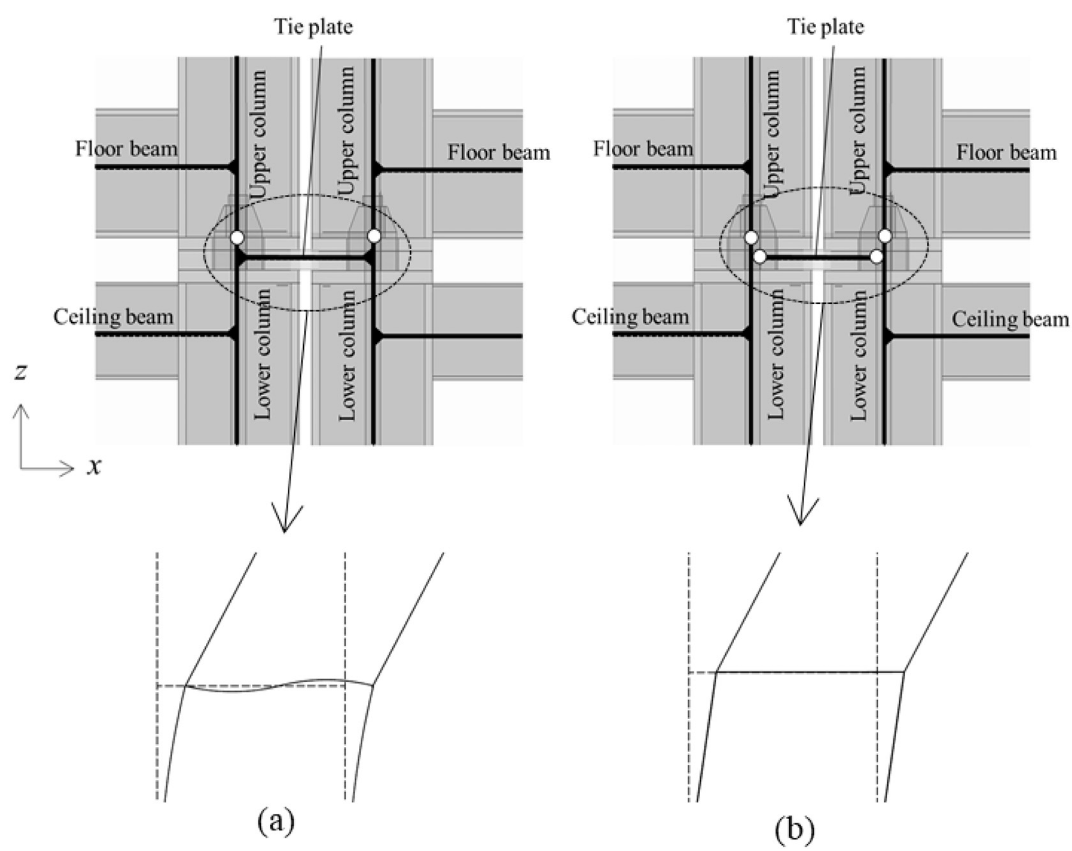


Fig. 19. Lateral deformed shape of joint models with pin-ended vertical connection while having horizontal connection modelled as (a) fix-ended (Case 2), and (b) pin-ended by releasing out-of-plane rotational restraint but fixing the in-plane rotational restraint (Case 3).

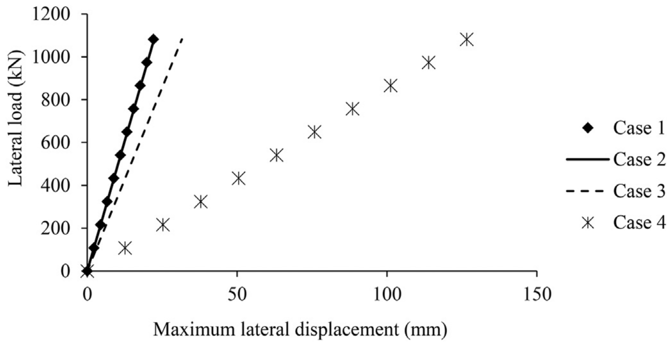


Fig. 20. Lateral force-displacement curves of 10-storey modular buildings with steel bracings with different inter-module joint models subject to wind load in *X*-direction.

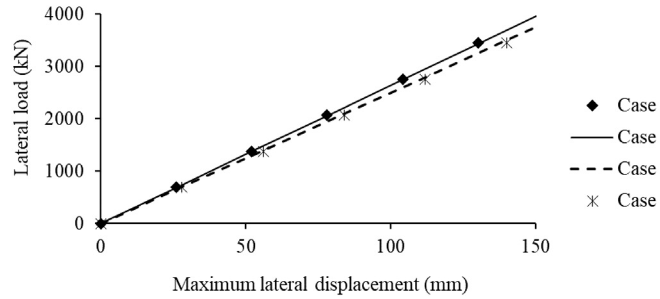


Fig. 22. Lateral force-displacement curve of 40-storey modular buildings with RC core wall modelled using discrete floor diaphragm with various joint models for wind load in *X*-direction.

does not make any difference if the floor slab is modelled as a rigid element or by using shell elements. The deformed shape is similar to that depicted in Fig. 24(b) as there are no excessive shear deformation between adjacent modular units under lateral load. It should be noted that a continuous rigid diaphragm disregards the deformation due to the contribution of the tie plate stiffness in resisting the lateral load. The entire floor is considered as one rigid body. This might result in unrealistic deformation of the modules that are connected only at the corner joints with flexible tie plates. Furthermore, the continuous rigid diaphragm assumption ignores the horizontal connections and hence it does not produce any force to be used for the joint design. Although having similar performance as the case with the slab modelled using shell elements, the discrete floor diaphragm model is too tedious to be applied as there are 550 floor diaphragms to be assigned in the modular building shown in Fig. 12. On the other hand, the case with a continuous semi-rigid diaphragm with wind load assigned on diaphragm stiffness centroid shows similar global response as the case with slab modelled using shell elements and wind load assignment on cladding.

Past study has reported that the diaphragm flexibility may cause additional inter-storey drift due to relative movement among the modules [12]. Fig. 26 shows the inter-storey drift of case 4 with different diaphragm assignments for the module slabs. It is found that when using a spring model, as proposed in this paper, the inter-storey drifts for cases with a continuous rigid diaphragm and discrete rigid diaphragms are close to the case with slab modelled using shell elements and wind loads assigned to the cladding. The inter-storey drifts of these cases pass the serviceability drift limit of 0.002 or 1/500 under wind load [35]. The case with a continuous semi-rigid diaphragm and wind load assigned on the diaphragm stiffness centroid may slightly underestimate the inter-storey drift by approximately 10%. As the wind load is applied at the diaphragm stiffness centroid, the load path transfer is different from the assignment on the perimeter cladding. Nonetheless, assigning a continuous semi-rigid diaphragm and wind load on diaphragm stiffness centroid may still be the most convenient and realistic approach to simulate the actual stiffness.

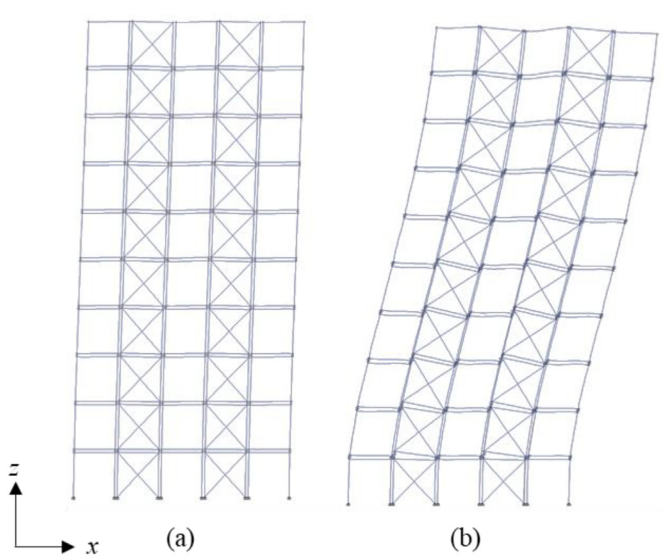


Fig. 21. Lateral deformed shape of 10-storey modular buildings with bracings under wind load *X*-direction for (a) pin-ended (releasing out-of-plane rotational restraint, fixing the in-plane rotational restraint) (i.e., Case 3), and (b) spring model (i.e., Case 4).

In summary, adopting an improved version of the simplified joint model with pin-ended vertical connection and releasing the out-of-plane rotational restraint of the horizontal tie plate while fixing its in-plane rotational restraint (Case 3) seems to be more appropriate in representing the actual behaviour of typical inter-module connections. The major difference between this simplified joint model and the spring model is the tensile stiffness and capacity of the vertical connection. In unbraced systems, the vertical connection is more critical and need to be considered properly in design. This is because the modules are required to resist the lateral load, hence the columns might be subjected to tension. Therefore, the use of the simplified joint model for the inter-module connection may not be able to predict the response behavior of unbraced modular buildings. For modules with steel bracings added to increase the lateral resistance of a modular building, the low tensile capacity of the tension rod and the enhanced rigidity of the laterally braced modules may amplify the toppling effect of the stacked-up modules. Proposing steel braced modules in modular buildings requires connection design that have comparable compression and tensile capacities such that the lateral force can be transmitted more efficiently to the braced modules. For modular building with the modules designed to resist the gravity loads and core wall to resist the lateral loads, assigning semi-rigid diaphragm is appropriate and convenient as the wind loads can be assigned on the diaphragm stiffness centroid, instead of creating artificial cladding for wind loads to be assigned on those cases with discrete diaphragms or shell elements for the slabs. However, proper design of the horizontal tie plate becomes crucial for buildings with RC core walls to avoid swing-out effect on the modules which are not directly connected to the core wall.

5. Conclusions

In this study, the effects of inter-module joint modelling and floor diaphragm modelling on global behaviour of high-rise modular buildings under lateral loads were investigated. To ensure that each module is stable on its own, the floor and ceiling beams must be rigidly connected to the columns. Two important connections were identified in steel modular buildings: (1) vertical connection that ties the upper and lower modules using a vertical rod, and (2) horizontal connection that ties the adjacent modules with a tie plate and shear keys. Simplified joint models were proposed to model these two types of connections. These connection models were then compared with the more "exact"

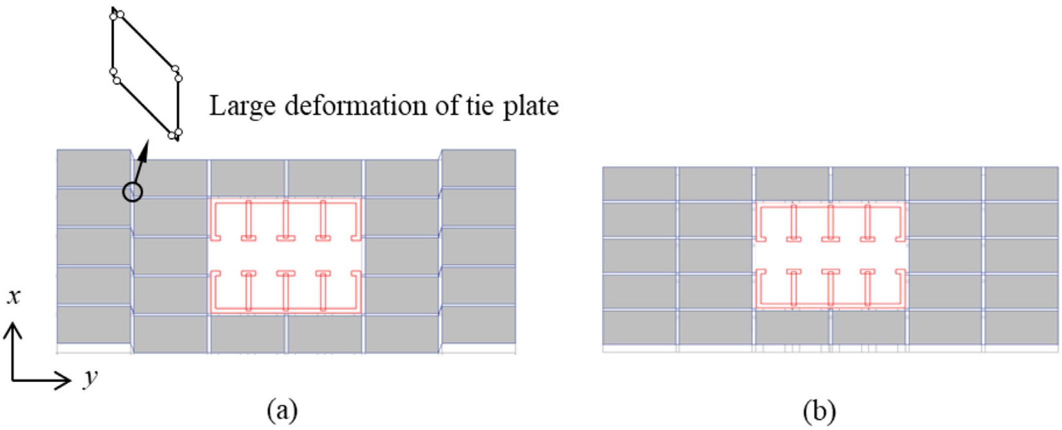


Fig. 23. Plan view of building deflection under wind load *X*-direction for case with (a) flexible tie plate and discrete rigid diaphragms, (b) enough capacity of tie plate.

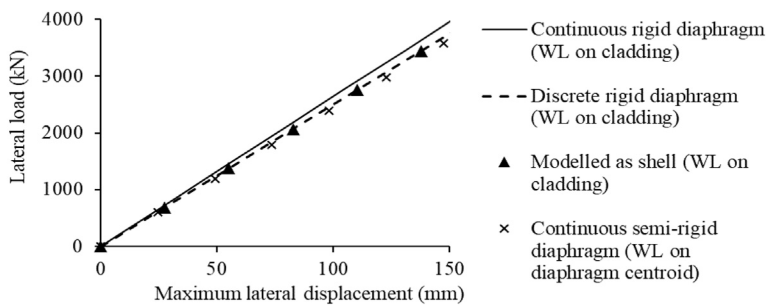


Fig. 24. Lateral force-displacement curve of 40-storey modular buildings with RC core wall and spring model (i.e., Case 4) while varying diaphragm assignments under wind load *X*-direction.

spring joint model that captures the actual stiffness of the inter-module connection so that proper guidance can be established regarding the use of simplified joint models for modular steel building. The following summarizes the key findings found in this study.

i. A translational spring model was developed and can be generalized for all inter-module connections by calculating the stiffness properties of the structural components in the connection. The joint model comprises of linear link and multilinear link elements that can be modelled easily using any established commercial software.
ii. In the simplified joint model, it is found that the column-to-column connection between the top and bottom modules can be assumed as pin-ended. Likewise, the boundary conditions of the horizontal tie plate may be modelled by releasing its out-of-plane rotational restraint (i.e., to prevent increased column stiffness) while fixing its in-plane rotational restraint (i.e., to account for in-plane shear stiffness). Such connection models can only be used when the modules are not designed to resist lateral load as the vertical tension stiffness of the connecting rod is not modelled.
iii. The horizontal tie plate becomes a critical component in resisting lateral loads especially when the modular building is braced by core walls and the modules are tied only at the four corners. Horizontal tie plate with sufficient thickness and in-plane shear stiffness should be provided to prevent swing-out effect causing excessive shear deformation between the modules that are not directly connected to the RC core walls.
iv. Adding steel bracings in the modular units does not help in increasing lateral resistance of a modular building when the column-to-column connection has relatively low tensile capacity and stiffness. The steel bracings enhance the lateral stiffness of the overall framework but also amplify the toppling effects of the stacked-up modules. For this reason, the tension rod in the column must have sufficient axial stiffness to prevent vertical separation of the columns and to be effective in transferring the lateral load to the braced modules.
v. Assigning semi-rigid diaphragms to all the modules at the storey level can capture the actual behaviour of the floor slab as compared to the case where the slabs are modelled using shell elements. The

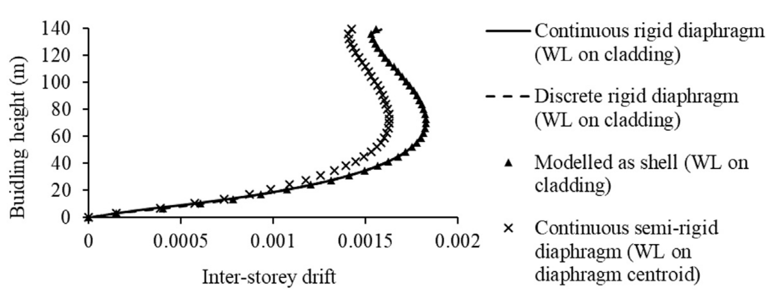


Fig. 25. Inter-storey drift of 40-storey modular buildings with RC core wall and spring model (i.e., Case 4) while varying diaphragm assignments under wind load *X*-direction.

wind load can be conveniently assigned on the diaphragm stiffness centroid instead of creating artificial cladding element to assign the lateral load. The corner joint model incorporating the effect of the tie plate was proposed for the analysis and design of high-rise modular buildings.

The proposed joint models were established based on the mechanical behavior of the connecting components in the joint. Further experimental work is needed to validate the performance of these joint models to provide confidence for practical implementation.

Declaration of Competing Interest

None.

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